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MODELS FOR RELIABILITY ASSESSMENT OF REINFORCED CONCRETE STRUCTURES IN CORROSIVE ENVIRONMENTS IN CONDITIONS OF A CHANGING CLIMATE

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Abstract. Reinforced concrete (RC) structures deteriorate with time and corrosion of reinforcing steel is one of the main causes for that. In the paper a framework for reliability-based assessment of durability of RC concrete structures in corrosive environments will be briefly described. Existing models for corrosion initiation, corrosion-induced cracking, and effects of corrosion on stiffness and strength of RC members will be considered. Special attention will be paid to the effects of a changing climate on corrosion risks to RC structures.

1 INTRODUCTION

Reinforced concrete (RC) structures deteriorate with time being subject to loads and aggressive, in particular corrosive, environments that affects their strength and serviceability. To ensure adequate performance and safety of RC structures during their service life the possible effects of deterioration should be considered in the design of new structures and assessment of existing ones. In the latter case, RC structures can be inspected or constantly monitored, especially when signs of deterioration have been detected. However, inspections and monitoring cannot provide complete information required for the assessment of structural condition. Moreover, to make a rational decision about what measures should be taken to ensure adequate safety of the structure its future performance over a certain period of time needs to be predicted. Thus, predictive models describing deterioration mechanisms and their structural effects are clearly needed. Taking into account numerous uncertainties associated with material properties, loads, environmental conditions, inspection/monitoring data, and the models themselves it has become increasingly accepted that such models should be probabilistic/stochastic and analysis/assessment of structures be reliability-based (e.g. [1]).

The initiation and rate of deterioration depends not only on material and structural properties, construction processes and imposed loads but also on the surrounding climatic

environment. Climate change modifies this environment due to: (i) gradual changes in average temperature, humidity, precipitation, atmospheric CO_2, etc.; (ii) larger variations of the environmental parameters during seasonal cycles and an increase in frequency and intensity of extreme weather. These phenomena may lead to the premature initiation of deterioration and an increase in damage risks [2].

In the paper a framework for the reliability-based assessment of durability of RC concrete structures in corrosive environments will be briefly described. Existing models for corrosion initiation, corrosion-induced cracking and effects of corrosion on stiffness and strength of RC members will be considered, including their applicability to probabilistic analysis in terms of data available for quantification of relevant uncertainties and required computational effort. Special attention will be paid to the potential effects of a changing climate on corrosion initiation and propagation in these structures.

2 DETERIORATION OF RC STRUCTURES IN CORROSIVE ENVIRONMENTS

There are a number of causes of deterioration of RC structures over time, e.g., alkali-aggregate reaction, sulphate attack, freeze-thaw cycles. However, the most common cause of deterioration is corrosion of embedded reinforcing steel due to carbonation or chloride contamination of concrete. The development of corrosion with time is usually considered as a two-phase process:

- corrosion initiation
  - penetration of aggressive agents (CO_2, chloride ions) through the concrete cover.
  - direct ingress of aggressive agents through cracks in the concrete cover.
- corrosion propagation
  - loss of cross-sectional area of reinforcing or prestressing steel.
  - cracking and then spalling/delamination of the concrete cover.
  - reduction of bond between concrete and reinforcing steel.
  - changes in ductility and mechanical properties of reinforcing/prestressing steel.

Typical development of corrosion-induced deterioration in a RC member over time is shown in Figure 1, where t_i denotes the time to corrosion initiation, t_{cr1} – the time of initial cracking of the concrete cover (i.e., hairline cracks of width less than 0.05 mm appear on the concrete surface), t_{cr} – the time of excessive cracking and spalling/delamination, and t_u – the time when an ultimate limit state is no longer satisfied due to strength reduction. It should be noted that the description of the deterioration process in Figure 1 is a schematic one, e.g., the deterioration rates within each stage are not necessarily constant and differences between the rates at different stages may be more significant.

The deterioration processes depend on a large number of influencing variables including concrete properties such as porosity, compression and tensile strength, water-cement ratio; thickness of the concrete cover; type of reinforcing steel; reinforcement spacing; quality of concrete constituents; exposure to aggressive agents (CO_2, chlorides); service loading; repair quality; etc. In addition, they depend on ambient environmental conditions, e.g., temperature, humidity, which may be affected in the future by climate change. Hence, in order to correctly predict corrosion-induced deterioration of new and existing RC structures during their intended service life it is essential to take effects of climate change into account. The deterioration processes are highly variable and there are numerous uncertainties associated

with the influencing variables and predictive models. For this reason probabilistic/stochastic models are employed to predict temporal and spatial effects of corrosion-induced deterioration on serviceability and strength of RC members.

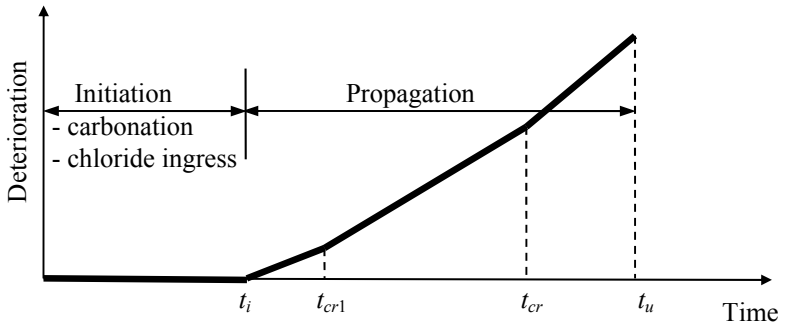


Figure 1: Schematic description of corrosion-induced deterioration in a RC member

3 RELIABILITY-BASED APPROACH TO DESIGN AND ASSESSMENT

Probabilistic/stochastic models are usually used within a reliability-based approach, when the condition of a structure or its member(s) is assessed in terms of the probability of failure, P_f, or related to it the reliability index, $\beta=-\Phi^{-1}(P_f)$, where Φ denotes the standard normal distribution function. For calculation of P_f uncertainties associated with material properties, loads, environmental conditions, models, etc. need to be quantified so that these parameters can be treated as random variables, processes or fields. The failure of a structure should then be formally defined and expressed as a function of relevant random variables, processes or fields, which is referred to as the limit state function, G. Conventionally, this function is defined such that $G\leq 0$ denotes failure of the structure, $G>0$ – its survival, while the boundary $G=0$ between the failure and safe domains is called the limit state surface. Thus, the limit state function can be expressed as

$$G(t,\mathbf{x},\boldsymbol{\theta}) = \mathbf{R}(t,\mathbf{x},\boldsymbol{\theta}) - \mathbf{S}(t,\mathbf{x},\boldsymbol{\theta}) \tag{1}$$

where **R** denotes capacity and **S** demand, e.g., the case of an ultimate limit state – resistance and load effect, respectively. Generally, **R** and **S** may include several random variables, processes or fields, which depend on time, t, and spatial coordinates $\mathbf{x}$. Very often, complete statistical information about random quantities is unavailable and certain assumptions (e.g., about their mean, variance and covariance) need to be made to describe them in terms of probabilistic models. The uncertainties associated with this can be represented by a vector of random parameters $\boldsymbol{\theta}$. Moreover, the development of deterioration in a structure may depend on loads acting on it, which means that **R** may depend on **S**.

For a given location within/on the surface of a structure, $\mathbf{x}_c$, the probability of failure during time interval $[0,t_c]$ is

$$P_f(t_c,\mathbf{x}_c,\boldsymbol{\theta}) = \Pr\left(G(t,\mathbf{x}_c,\boldsymbol{\theta}) \leq 0 \,\middle|\, t\in[0,t_c]\right) \tag{2}$$

This probability depends on values of the parameters $\boldsymbol{\theta}$ while for decision making a unique definition of the probability of failure is needed. The latter can be obtained as the expected value of Eq. (2)

$$P_f(t_c, \mathbf{x}_c) = E\left(P_f(t_c, \mathbf{x}_c, \boldsymbol{\theta})\right) = \int_{\boldsymbol{\theta}} P_f(t_c, \mathbf{x}_c, \boldsymbol{\theta}) f_{\boldsymbol{\Theta}}(\boldsymbol{\theta}) d\boldsymbol{\theta} \tag{3}$$

where $E(\cdot)$ is the expectation operator and $f_{\boldsymbol{\Theta}}(\boldsymbol{\theta})$ is the joint probability density of the parameters $\boldsymbol{\theta}$. A decision is then made by comparing the obtained P_f (or β) with the corresponding target value. The uncertainty associated with parameters $\boldsymbol{\theta}$ is usually the highest at the design stage, when the structure does not yet exist so that material properties or loads cannot be actually measured. For existing structures additional information can be collected by on-site inspections and then used for updating probabilistic models based on a Bayesian approach.

4 CORROSION INITIATION

4.1 Carbonation induced corrosion

Carbonation is a process involving two mechanisms – diffusion of carbon dioxide, CO_2, from the atmosphere into concrete and its reaction with the alkaline cement hydration products. The reaction leads to reduction of the alkalinity, pH, of the concrete pore solution from 12-13 to below 9 that then destroys the thin passive oxide layer on the surface of reinforcing steel and makes the steel susceptible to corrosion. If to assume that diffusion of CO_2 into concrete is a steady state process described by Fick's 1st law of diffusion and the reaction with the alkaline cement hydration products occurs instantaneously then the mass balance equation at the carbonation front can be expressed as

$$b dx = D_{CO_2} \frac{C_{CO_2}}{x} dt \tag{4}$$

where D_{CO2} is the diffusion coefficient of CO_2 in concrete, C_{CO2} the ambient concentration of CO_2, b the CO_2-binding capacity of concrete, t the time in service and x the distance from the concrete surface to the carbonation front. Note that the steady state formulation means that D_{CO2} and C_{CO2} do not change with time. If D_{CO2} also does not depend on x, the solution of Eq. (4) is

$$x_c(t) = \sqrt{\frac{2 D_{CO_2} C_{CO_2}}{b}} \sqrt{t} \tag{5}$$

where $x_c(t)$ is the depth of the carbonation front at time t.

However, both D_{CO2} and the rate of reaction between CO_2 and the alkaline cement hydration products strongly depend on the pore relative humidity, RH, of concrete. Since the pore RH depends on the ambient relative humidity that changes with time, D_{CO2} is time-variant as well. The diffusion of CO_2 also depends on other time-variant parameters like the degree of hydration and temperature. To account for these influences a number of semi-empirical models, manly based on Eq. (5), have been proposed [3].

In order to use a model for probabilistic analysis quantitative description of uncertainties associated with its parameters and the model itself, or at least sufficient statistical data for doing that, is needed. One of semi-empirical models that satisfies this requirement is presented in [4]. The model was used in DuraCrete [5] and LIFECON [6], where uncertainties

associated with its parameters were quantified. According to this model the depth of carbonation at time t is evaluated as

$$x_c(t) = \sqrt{2k_e k_c R_{NAC,0}^{-1} C_{CO_2,s}}\sqrt{t}W(t) \tag{6}$$

where k_e is the environmental function which accounts for the moisture conditions, k_c the execution transfer parameter, $C_{CO2,s}$ the concentration of CO_2 in the surrounding air, $W(t)$ the weather function and $R^{-1}_{NAC,0}$ the inverse carbonation resistance of concrete determined in natural conditions that represents both the diffusivity and the binding capacity of concrete

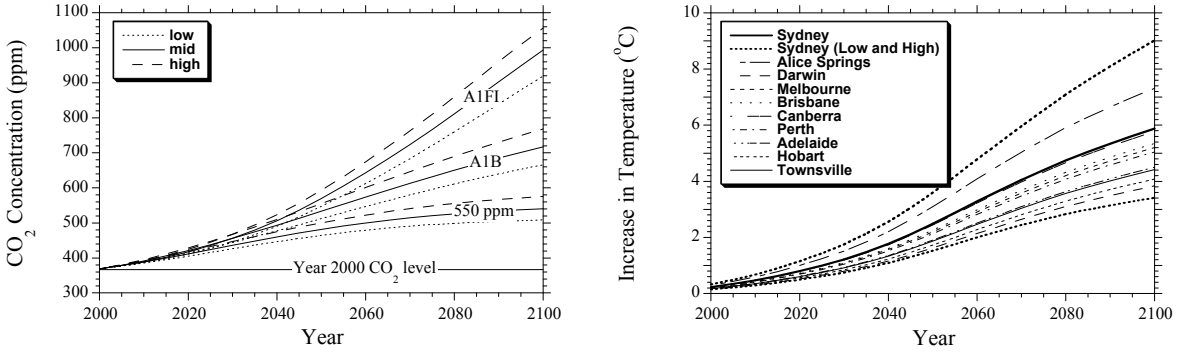
$$R_{NAC,0}^{-1} = \frac{D_{CO_2}}{b} \tag{7}$$

The probability of corrosion initiation, P_{corr}, is time-dependent and can be expressed as

$$P_{corr}(t) = \Pr[c - x_c(t) < 0] \tag{8}$$

where c is the thickness of the concrete cover, which should also be treated as a random variable. More information about the model parameters, including their statistical description, can be found in [4]. Another semi-probabilistic model of carbonation, for which statistical description of its parameters is provided, has been recently presented in [7].

Climate change leads to an increase over time of environmental variables controlling carbonation such as the atmospheric concentration of CO_2 and temperature and possible changes in other important environmental parameters, e.g., humidity. For example, Figure 2(a) shows projected increases in the CO_2 concentration during this century for three carbon emission scenarios: A1FI, A1B and 550 ppm, i.e., high, medium and CO_2 stabilisation at 550 ppm by 2150 scenarios, respectively. The results are presented in terms of mean values and low and upper bounds, which account for CO_2 projection modelling errors. Figure 2(b) shows predicted increases in the mean temperature for nine Australian cities, for the A1FI emission scenario using CSIRO Mk3.5 Climate Model. In addition, to illustrate uncertainties associated with the temperature prediction the figure shows low and high bounds for Sydney based on nine General Circulation Models (see [8,9] for more details).



(a) (b)

Figure 2: (a) Projected low, mean and high estimates of CO_2 concentrations; (b) Predicted increases in mean temperature and their low and high bounds for Sydney for the A1FI emission scenario.

Effects of climate change on carbonation cannot be taken into account within a steady state formulation since the main parameters such as C_{CO2} and D_{CO2} are continuously changing with time. Thus, the solution given by Eq. (7) and semi-empirical models based on it are not directly applicable. As an approximation, the considered time period can be divided into intervals and then assumed that within each interval C_{CO2} and D_{CO2} remain constant, i.e., the continuous functions shown in Figure 2 are approximated by step ones. A model based on the steady state formulation is then applied separately to each time interval and the obtained results are aggregated. In order to obtain a reasonably accurate solution the number of time intervals should be sufficiently large. For more accurate results the diffusion of CO_2 into concrete should be modelled by Fick's 2nd law of diffusion.

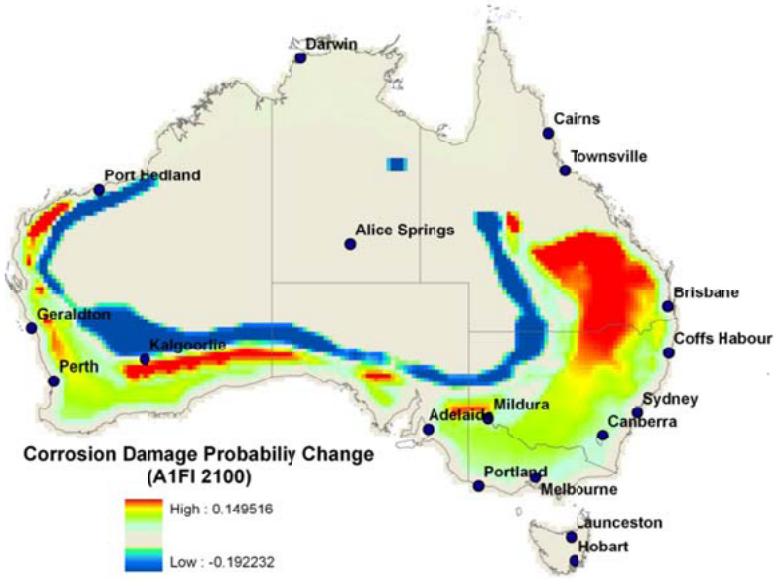


Figure 3: Change in probability of carbonation-induced corrosion damage by 2100 for A1FI scenario [9]

Using the DuraCrete model for carbonation [5] and the approximated approach described above, Stewart et al. [9] estimated changes in the probability of structural damage (excessive cracking) due to carbonation-induced corrosion for various regions in Australia over the period from 2000 to 2100 for the A1FI emission scenario. Results are shown in Figure 3 and take into account changes in the times of corrosion initiation and subsequent excessive cracking of the concrete cover. According to the results, under a changing climate the probability of carbonation-induced corrosion damage can, depending on the region, be 19% below the baseline (i.e., 2000) to 15% above the baseline by 2100. In practical terms, this is equivalent to expecting that an additional 15% of every RC member in the worst affected regions will be damaged and in need of maintenance or repair by 2100. Reductions in damage risks occur mainly in inland Australia (where there is little infrastructure) due to reductions in humidity. The probability is higher around the boundary between the arid climatic zone in central Australia and the temperate climatic zone in the west, south as well as east of Australia. This is mostly caused by a current lower cover requirement for RC structures in arid and temperate climatic zones.

4.2 Chloride induced corrosion

Chloride-induced corrosion is generally recognised as more dangerous for RC structures compared to that induced by carbonation. Corrosion of this type starts when the chloride concentration near reinforcing steel exceeds a certain threshold value. Since the initial concentration of chlorides in reinforced concrete must be very low the time of corrosion initiation is mainly controlled by ingress of chloride ions from external sources (e.g., seawater, de-icing salts).

The ingress of chloride ions into concrete is a complex process involving different transport mechanisms such as ionic diffusion and convection (mainly in the form of capillary suction). As such it depends on a large number of parameters, e.g., composition of concrete, its porosity and microstructure, degree of pore saturation, temperature, exposure conditions. A number of these parameters are inter- and time-dependent, in particular due to chemical reactions which proceed in concrete over time. Since only free chloride ions can penetrate into concrete the chloride ingress depends also on binding of chlorides with the cement paste hydration products, which in its turn depends on the chloride concentration.

Physically based models with various levels of complexity have been proposed to describe this process. The models can be broadly divided into those, which take into account the interaction of chlorides with other ions present in the concrete pore solution and those which neglect this phenomenon. Models from the first group are usually based on the so-called multi-species approach with the flux of different ions described by the Nernst-Planck equation (e.g., [10]). Due to their complexity and lack of data for estimation of their parameters, in particular uncertainties associated with these parameters, the models are currently not used in practice and not suitable for probabilistic analysis. Models from the second group usually take into account chloride binding, coupled chloride and moisture diffusion and some of them also heat transfer (e.g., [11]). Although these models are simpler than those from the first group they are still rather complex and available experimental data are still insufficient for evaluation of their parameters, in particular quantification of their uncertainties, that limits their applicability within a reliability-based framework. Val and Trapper [12] have proposed a relatively simple probabilistic model belonging to the second group, which takes into account chloride binding and coupled chloride and moisture diffusion and can describe both one- (1-D) and two-dimensional (2-D) ingress of chloride ions. The model has been extended in [13] to account for heat transfer. Such models are very suitable for considering effects of climate change [14]; however, quantification of uncertainties associated with their parameters is still an unresolved issue that casts doubts on the validity of obtained results.

Models usually used in practice are semi-empirical models based on Fick's 2nd law of diffusion since it has been observed that chloride profiles (at least for internal layers of concrete where the influence of convection is insignificant) may be approximated sufficiently well by solutions of the following differential equation representing this law

$$\frac{\partial C}{\partial t} = D_c \frac{\partial^2 C}{\partial x^2} \tag{9}$$

where D_c is the chloride diffusion coefficient, C the chloride concentration, and x the distance from the concrete surface. Initially, in order to obtain a solution of Fick's 2nd law it was typical to assume that D_c and the surface chloride concentration, C_s, were time-invariant

constants. However, experimental evidence has been collected indicating that D_c and C_s are time-dependent and so a number of empirical models taking into account the time-dependency of D_c or C_s have been proposed (e.g., [15,16]).

Time-dependence of the chloride diffusion coefficient is usually described by a power function [4-6]

$$D_a(t) = D_{a,ref}\left(\frac{t_{ref}}{t}\right)^m \tag{10}$$

where D_a denotes the apparent diffusion coefficient, $D_{a,ref}$ its value at the reference time t_{ref}, which also depends on temperature, and m the age factor. It is important to stress that according to its definition D_a remains constant within a time period t and changes only when the duration of the time period changes. Therefore, D_a does not represent a physical property of concrete at a particular moment in time, but rather characterises an average diffusivity over the time period t. Results are very sensitive to values of the age factor [17].

Assuming that the surface chloride concentration remains constant with time and the time-dependence of the diffusion is described by Eq. (10) the solution of Eq. (9) can be expressed as (e.g., [4-6])

$$C(x,t) = C_i + \left(C_{s,\Delta x} - C_i\right)\mathrm{erfc}\left(\frac{x-\Delta x}{2\sqrt{tD_a(t)}}\right) \tag{11}$$

where Δx is the depth of the convection zone (i.e., the surface concrete layer in which chloride penetration is mainly governed by convection and Fick's 2nd law is inapplicable), $C_{s,\Delta x}(t)$ the chloride concentration at depth Δx, $C_i(x)$ the initial chloride concentration and erfc the error function complement. The solution is simple and statistical description of its parameters can be found in [4]. Thus, it is suitable for probabilistic analysis and since the apparent diffusion coefficient depends on temperature it can also be used to examine effects of climate change. It should be noted that this solution is applied under the assumption that the concrete cover is uncracked. However, cracks have a significant influence on the chloride diffusion coefficient and, subsequently, on t_i (e.g., [18]).

The probability of corrosion initiation is time-dependent and can be estimated as

$$P_{corr}(t) = \Pr\left[C_{crit} - C(x=c,t) < 0\right] \tag{12}$$

where C_{crit} is the threshold chloride concentration, which is a highly variable parameter. Its values reported by different authors vary significantly and currently there is even no agreement on how to present it – as a total or free chloride content, expressed relative to the weight of cement or concrete, or as the chloride to hydroxyl concentration ratio [19,20].

In the context of climate change, results obtained for Australia indicate that chloride induced corrosion initiation is much less sensitive to climate change than carbonation induced corrosion. An increase in the risk of chloride induced corrosion over the period from 2000 to 2100 for the A1FI emission scenario is 0 to 3.2%, depending on the region [9] (see Figure 4). However, the proportional increase in damage risks can easily exceed 10%. This lower change in risk profile may be due to the fact that under climate change temperature rises in coastal areas are likely to be much less marked than those occurring inland.

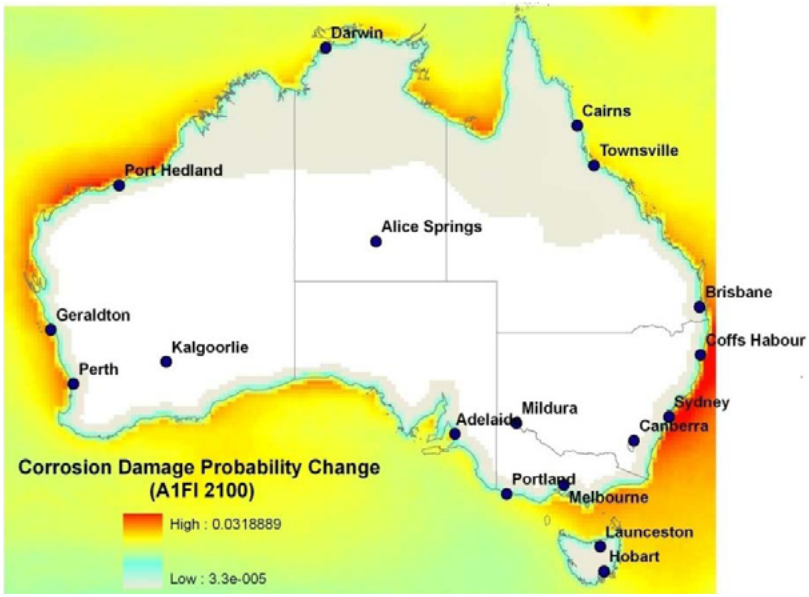


Figure 4: Change in probability of chloride-induced corrosion damage by 2100 for A1FI emission scenario [9]

5 CORROSION PROPAGATION

5.1 Corrosion rate

To predict the development of corrosion-induced deterioration with time the rate of corrosion needs to be known. The latter is usually described in terms either of the corrosion current density, i_{corr}, or the corrosion penetration, V_{corr}. A number of empirical models predicting i_{corr} or V_{corr} have been proposed (e.g., [21]). Another group of models relates i_{corr} (or V_{corr}) with either electrical resistivity of concrete (e.g. [5,6]) or its oxygen diffusivity [18]. Finally, there are physically based electrochemical models which consider the polarization behaviour of anodic and cathodic regions on the steel surface. These are complex models, for most of which solutions can only be found numerically [22]. However, there are currently insufficient experimental data for estimation of the model parameters, in particular of the spatial distribution (dimensions, locations and geometry) of anodic and cathodic regions on the steel surface.

For probabilistic analysis a model presented in [5,6], which relates V_{corr} with the concrete resistivity, $\rho(t)$, can be employed

$$V_{corr} = \frac{m_0}{\rho(t)} F_{Cl} w_t \ \text{[μm/year]} \tag{13}$$

where m_0=882 μm·Ωm/year and F_{Cl}=1+k(C-C_{crit}) the chloride corrosion rate factor ($F_{Cl} \geq 1$). The concrete resistivity is time-variant and depends on a number of factors

$$\rho(t) = \rho_0 k_{R,T} k_{R,RH} k_{R,Cl} \left(\frac{t}{t_0}\right)^n \tag{14}$$

where ρ_0 is the specific electrical resistivity of concrete at time t_0, $k_{R,T}$= exp[$b_{R,T}$(1/T_{real}-1/T_0)]

the temperature factor, $b_{R,T}$ the regression variable, T_0=293 K, $k_{R,RH}$ the relative humidity factor, $k_{R,Cl}$=1-(1-a_{Cl})/2 the chloride factor, t_0=28 days, t the age of concrete (in analysis $t \leq 1$ year), and n the age factor for resistivity. In analysis k, ρ_0, $b_{R,T}$, $k_{R,RH}$, $k_{R,Cl}$ (or a_{Cl}), and n are treated as random variables, whose statistical properties are given in [5,6].

5.2 Loss of cross sectional area of reinforcing steel

Two types of corrosion – general and pitting – are possible. General corrosion affects a substantial area of reinforcement with more or less uniform metal loss over the perimeter of reinforcing bars. For this type of corrosion the cross-sectional area of a reinforcing bar with the original diameter d after t years since corrosion initiation will be

$$A_s(t) = \frac{\pi\left[d - 2\times 10^3 V_{corr} t\right]^2}{4} \geq 0 \quad [\text{mm}^2] \tag{15}$$

In contrast to general corrosion, pitting corrosion concentrates over small areas of reinforcement. The depth of a pit, p_{max}, representing the maximum penetration of pitting after t year since corrosion initiation can be evaluated as

$$p_{\max}(t) = V_{corr} \times 10^3 tR \quad [\text{mm}] \tag{16}$$

where R=p_{max}/p and p is the average penetration of general corrosion. According to available experimental data, R is highly variable and its uncertainty can be described by the Gumbel distribution [23]. The cross-sectional area of a pit can be estimated based on hemispherical model of a pit proposed by Val and Melchers [24]. Pitting corrosion may also affect strength and ductility of reinforcing steel [25].

5.3 Cover cracking

Corrosion products occupy a larger volume than the consumed steel. As they form, they exert pressure on the concrete surrounding a corroding reinforcing bar that eventually leads to cracking of the concrete cover. The latter affects mainly serviceability of RC structures. A large number of empirical, analytical and numerical models have been proposed to estimate the time, Δt_{cr1}, between corrosion initiation, t_i, and that of initial cracking of the concrete cover, t_{cr1}. Based on a critical review of the models provided in [26], the so-called thick-walled cylinder model can be recommended for probabilistic analysis. According to this model, the time between corrosion initiation and cover cracking can be estimated as

$$\Delta t_{cr1} = \frac{c f_{ct}}{(1-\alpha_v)(1-\eta)E_{c,ef}V_{corr}}\left[1 + \nu_c + \frac{d^2}{2c(c+d)}\right] \quad [\text{years}] \tag{17}$$

where ν_c is the Poisson's ratio of concrete, $E_{c,ef}$ the effective modulus of elasticity of concrete, f_{ct} the tensile strength of concrete, α_v is the volumetric expansion ratio of corrosion products and η the relative fraction of the corrosion products diffused into concrete pores and cracks. In probabilistic analysis the parameters α_v and η can be modelled by a Beta distribution with means 3.0 and 0.7 and coefficient of variation of 0.3 defined on [2.,6.4] and [0.,0.9], respectively.

5.4 Reduction in bond

Corrosion influences a number of parameters, which control the interaction between concrete and reinforcing steel, such as the level of confinement, adhesion and friction between concrete and steel and the height of the ribs in deformed bars that decreases as corrosion propagates. These affect both strength and stiffness of the bond between reinforcing steel and concrete. For probabilistic analysis an empirical model proposed in [27], which relates the normalised bond strength with the average corrosion penetration, p, can be used

$$\frac{\tau_{\max}}{\tau_{\max,0}} = \begin{cases} 1 + (k_1 - 1)\dfrac{p}{p_{cr1}}, & p \le p_{cr1} \\ \max\left[k_1 - k_2(p - p_{cr1}); 0.15\right], & p > p_{cr1} \end{cases} \tag{18}$$

where $\tau_{\max,0}$ is the initial bond strength, $\tau_{\max}$ the bond strength with corrosion, p_{cr1} the corrosion penetration corresponding to crack initiation in the concrete cover, and k_1 and k_2 are the coefficients which depend on the level of confinement provided by concrete and stirrups. The model has been calibrated using several sets of experimental data from different sources and statistical description of its parameters is provided.

6 CONCLUSIONS

The paper has presented an overview of the models that can be used in probabilistic analysis to assess the reliability of RC structures deteriorating due to corrosion of reinforcing steel. The applicability of the models to evaluation of the impact of climate change is also discussed. Examples illustrating the effects of climate change on the risk of corrosion-induced damage of RC structures in Australia are provided.

REFERENCES

[1] Val, D.V. Deterioration of strength of RC beams due to corrosion and its influence on beam reliability. *J. Struct. Eng*, *ASCE* (2007) **133**:1297-1306.
[2] Stewart, M.G., Wang, X. and Nguyen, M.N. Climate change impact and risks of concrete infrastructure deterioration. *Eng. Struct* (2011) **33**:1326-1337.
[3] DuraCrete – Modelling of Degradation, Document BE95-1347/R4-5, The European Union – Brite EuRam III (1998).
[4] ModelCode for Service Life Design. *fib* Bulletin 34, *fib*, Lausanne (2006).
[5] DuraCrete – Final Technical Report. General Guidelines for Durability Design and Redesign, Document BE95-1347/R17, The European Union – Brite EuRam III (2000).
[6] LIFECON Deliverable D 3.2: Service Life Models. European Community: Fifth Framework Program (2003).
[7] Thiery, M., Cremona, C. and Baroghel-Bouny, V. Application of the reliability theory to the assessment of carbonation-induced corrosion risk of rebars. *European J. Environ. Civil Eng* (2012) **16**:273-287.
[8] Stewart, M.G., Wang, X.M. and Nguyen, M.N. Climate change impact and risks of concrete infrastructure deterioration. *Eng. Struct* (2011) **33**:1326-1337.
[9] Stewart, M.G., Wang, X. and Nguyen, M.N., Climate change adaptation for corrosion control of concrete infrastructure. *Struct. Saf* (2012) **35**:29-39.
[10] Khitab, A., Lorente, S. and Ollivier, J.P. Predictive model for chloride penetration

through concrete. *Mag. Concr. Res* (2005) **57**:511-520.
[11] Meijers, S.J.H., Bijen, J.M.J.M., de Borst, R. and Fraaij, A.L.A. Computational results of model for chloride ingress in concrete including convection, drying-wetting cycles and carbonation. *Mater. Struct* (2005) **38**:145-154.
[12] Val, D.V. and Trapper, P.A. Probabilistic evaluation of initiation time of chloride-induced corrosion. *Reliab. Eng. Syst. Saf* (2008) **93**:364-372.
[13] Bastidas-Arteaga, E., Chateauneuf, A., Sánchez-Silva, M., Bressolette, P. and Schoefs, F. A comprehensive probabilistic model of chloride ingress in unsaturated concrete. *Eng. Struct* (2011) **33**:720-730.
[14] Bastidas-Arteaga, E., Schoefs, F., Stewart, M.G. and Wang, X. Influence of global warming on durability of corroding RC structures: A probabilistic approach. *Eng. Struct* (2013) **51**:259-266.
[15] Maage, M., Helland, S., Poulsen, E., Vennesland, Ø. and Carlsen, J.E. Service life prediction of existing concrete structures exposed to marine environment. *ACI Mater. J* (1996) **93**:602-608.
[16] Ann, K.Y., Ahn, J.H. and Ryou, J.S. The importance of chloride content at the concrete surface in assessing the time to corrosion of steel in concrete structures. *Constr. Build. Mater* (2009) **23**:239-245.
[17] Val, D.V. Service-life performance of RC structures made with supplementary cementitious materials in chloride-contaminated environments. Int. RILEM-JCI Seminar on Concrete Durability and Service Life Planning (ConcreteLife'06), Ein-Bokek, Israel, (2006) 363-373.
[18] Takewaka, K., Yamaguchi, T. and Maeda S. Simulation model for deterioration of concrete structures due to chloride attack. *J. Adv. Concr. Technol* (2003) **1**:139-146.
[19] Alonso, C., Andrade, C., Castellote, M. and Castro P. Chloride threshold values to depassivate reinforcing bars embedded in a standardized OPC mortar. *Cem. Concr. Res* (2000) **30**:1047-1055.
[20] Angst, U., Elsener, B., Larsen, C.K. and Vennesland, Ø. Critical chloride content in reinforced concrete – A review. *Cem. Concr. Res* (2009) **39**:1122-1138.
[21] Vu, K.A.T. and Stewart, M.G. Predicting the likelihood and extent of RC corrosion-induced cracking. *J. Struct. Eng, ASCE* (2005) **131**:1681-1689.
[22] Redaelli, E., Bertolini, L., Peelen, W. and Polder, R. FEM-models for the propagation period of chloride induced reinforcement corrosion. *Mater. Corros* (2006) **57**:628-635.
[23] Stewart, M.G. Spatial variability of pitting corrosion and its influence on structural fragility and reliability of RC beams in flexure. Struct. Saf (2004) **26**:453-470.
[24] Val, D.V. and Melchers, R.E. Reliability of deteriorating RC slab bridges. *J. Struct. Eng, ASCE* (1997) **123**:1638-1644.
[25] Stewart, M.G. Mechanical behaviour of pitting corrosion of flexural and shear reinforcement and its effect on structural reliability of corroding RC beams. *Struct. Saf* (2009) 31:19-30.
[26] Chernin, L. and Val, D.V. Prediction of corrosion-induced cover cracking in reinforced concrete structures. *Constr. Build. Mater* (2011) **25**:1854-1869.
[27] Val, D.V. and Chernin, L. Serviceability reliability of reinforced concrete beams with corroded reinforcement. *J. Struct. Eng*, *ASCE* (2009) **135**:896-905.